Chapter 2
Open Channel Hydraulic Theory

2-1. Physical Hydraulic Elements

a. General. The physical hydraulic elements concerned in hydraulic design of channels consist of invert slope (S_o), cross-sectional area (A), wetted perimeter (P), and equivalent boundary surface roughness (k). The hydraulic radius (R) used in resistance formulae is the ratio A/P . The invert slope of proposed channel improvement is controlled primarily by elevations of the ground along the alignment as determined by preliminary layout discussed in paragraph 1-6d. A center-line profile between controlling elevations along the proposed alignment will indicate a preliminary channel slope.

b. Channel cross section.

(1) The proper channel cross section for a given reach is the one that has adequate hydraulic capacity for a minimum cost of construction and maintenance. The economics must include the costs of right-of-way and structures such as bridges. In rural areas a trapezoidal cross section may be least costly, whereas in urban areas a rectangular cross section is often the least costly.

(2) Plate 1[1] shows a sample cost computation and related cost curve for a reach of curved rectangular concrete channel. Similar procedures may be applied to compute the cost for any type of cross section considered for design. Special types of concrete channel cross sections are shown in Plate 2: the V-bottom channel and the modified trapezoidal channel. The latter has a small low-flow channel in the center.

(a) In the V-bottom channel, low flows are concentrated along the channel center line. This prevents moderate flow from meandering over the entire channel width, which would result in random deposition of material across the invert as in the case of a horizontal bottom. Deposition in the center of the V-bottom is removed by larger flows. Because the wear caused by bed load is also concentrated near the center line, maintenance cost is reduced.

(b) In the modified trapezoidal cross section, vertical sidewalls reduce the top width. This design is desirable when the width of the right-of-way is limited. A small, low-flow channel in the center of the cross section provides a flow way into which subdrainage can be emptied. In cold climates, the low-flow channel reduces the invert area subjected to the deleterious effects of freezing and thawing. In some cases the low-flow channel may serve as a fishway.

c. Roughness. The concept of surface roughness as the basic parameter in flow resistance (friction) is almost universally accepted. Absolute roughness is determined from the physical dimensions of the wetted surface irregularities and is normally of theoretical interest only. Equivalent roughness is a linear dimension (effective roughness height) directly related to the boundary resistance of the channel (Plate 3). The relations between roughness and the various coefficients for friction formulae are adequately covered by Chow (1959, chap 8). * Friction formulae and their uses are discussed in paragraph 2-2, and methods for predicting Manning's roughness coefficient n are discussed in Chapter 5. *

d. Composite roughness. Where there is material variation in roughness between various portions of the wetted perimeter such as might be found in natural channels or channels with protected banks and natural inverts, an equivalent or effective roughness or friction coefficient for each stage considered should be determined. Appendix C illustrates a method for determining a composite value of k for each stage. Plates 4 and 5 give the relation between k and Manning's n for flows in the rough flow zone shown in Plate 3. HDC sheets 631-4 and 631-4/1 also give a procedure for determining an effective Manning's n .

e. Hydraulic efficiency. The problem of the most efficient cross section is treated by Brater and King (1976, see pp 7-5 to 7-7) and Chow (1959, see paragraph 7-6).

2-2. Hydraulic Design Aspects

a. General. This presentation assumes that the design engineer is fully acquainted with the hydraulic theories involved in uniform and gradually varied flows, steady and unsteady flows, energy and momentum principles, and other aspects such as friction related to hydraulic design normally covered in hydraulic texts and handbooks such as those by Brater and King (1976) and Chow (1959). The following is presented as guidance in the method of application of textbook material and to give additional information not readily available in reference

[1] Plates mentioned in this and succeeding chapters are included in Appendix B as Plates B-1, B-2, etc.

material. The use of k is emphasized herein because computational results are relatively insensitive to errors in assigned values of k. However, use of Manning's n has been retained in several procedures because of its wide acceptance and simplicity of use. This applies particularly to varied flow profiles, pulsating flow, and the design of free-surface hydraulic models.

b. Friction losses.

(1) The importance that friction plays in the determination of flow characteristics in channels cannot be overstressed. Three equations (Chezy's, Manning's, and Darcy's) are in general use for the determination of losses due to friction. These equations expressed as friction slope S_f, i.e., slope of the energy grade line, are

(a) Chezy:

$$S_f = \frac{V^2}{C^2 R} \tag{2-1}$$

(b) Manning:

$$S_f = \frac{V^2 n^2}{2.21 R^{4/3}} \tag{2-2}$$

(c) Darcy:

$$S_f = \frac{f V^2}{8 R g} \tag{2-3}$$

where

V = velocity

C = Chezy coefficient

f = Darcy-Weisbach resistance coefficient

g = acceleration of gravity

The relation between the coefficients in these equations can be expressed as

$$\frac{C}{1.486} = \frac{R^{1/6}}{n} = \frac{10.8}{f^{1/2}} \tag{2-4}$$

(2) When determining friction coefficients, it should be recognized that the energy grade line and therefore the friction coefficient include uniformly occurring turbulence and eddy losses as well as the friction loss. Equivalent roughness for the same reason. Special, locally occurring turbulence and eddy losses are to be determined separately as covered in hydraulic textbooks and elsewhere in this manual.

c. Friction coefficients.

(1) The equations for using equivalent roughness to determine friction coefficients (Plate 3) are

(a) For hydraulically smooth channels

$$C = 32.6 \log_{10}\left(\frac{5.2 R_n}{C}\right) \tag{2-5}$$

(b) For hydraulically rough channels

$$C = 32.6 \log_{10}\left(\frac{12.2 R}{k}\right) \tag{2-6}$$

where R_n is the Reynolds number.

(2) For the channel surface to be hydraulically smooth, the equivalent roughness must be less than the critical value given by paragraph 8-12 of Chow (1959).

$$k_c = \left(\frac{5C}{\sqrt{g}}\right)\left(\frac{\nu}{V}\right) \tag{2-7}$$

where ν is the kinematic viscosity of water.

(3) Most channels (including concrete-lined channels) with appreciable velocity are hydraulically rough. Plates 4 and 5 are furnished as an aid for determining friction coefficients from equivalent roughness. Irrigation and power canals generally fall in the transition zone shown in Plate 3.

(4) Table 2-1, extracted from HDC sheets 631 to 631-2, provides acceptable equivalent roughness values for straight, concrete-lined channels.

(5) See Chapter 3 for friction coefficients for riprap.

(6) Values of k for natural river channels usually fall between 0.1 and 3.0 ft (see Table 8-1 of Chow 1959). These values will normally be much larger than the spherical diameters of the bed materials to account for boundary irregularities and sand waves. When friction coefficients can be determined from experienced flow information, k values should then be computed using the relations described in Equation 2-6. The k values so determined apply to the surfaces wetted by the experienced flows. Additional wetted surfaces at higher stages should be assigned assumed k values and an effective roughness coefficient computed by the method outlined in Appendix C if the increased wetted surfaces are estimated to be appreciably smoother or rougher. Values of k for natural channels may also be estimated from Figures 8 and 9 of Chow (1959) if experimental data are not available.

Table 2-1
Acceptable Equivalent Roughness Values

Design Problem	k , ft
Discharge Capacity	0.007
Maximum Velocity	0.002
Proximity to Critical Depth[1]	
Tranquil Flow	0.002
Rapid Flow	0.007

Note:
1. To prevent undesirable undulating waves, ratios of flow depth to critical depth between 0.9 and 1.1 should be avoided where economically feasible.

d. Flow classification. There are several different types of flow classification. Those treated in this paragraph assume that the channel has a uniform cross-sectional rigid boundary. The concepts of tranquil and rapid flows are discussed in (1) below. The applicability of the newer concepts of steady rapid flow and pulsating rapid flow to design problems are treated in (2) below. All of these concepts are considered from the viewpoint of uniform flow where the water-surface slope and energy grade line are parallel to the bottom slope. Flow classification of nonuniform flow in channels of uniform solid boundaries or prismatic channels is discussed in (3) below. The design approaches to flow in nonprismatic channels are treated in other portions of this manual.

(1) Tranquil and rapid flows.

(a) The distinction between tranquil flow and rapid flow involves critical depth. The concept of specific energy H_e can be used to define critical depth. Specific energy is defined by

$$H_e = d + \alpha \frac{V^2}{2g} \tag{2-8}$$

where

d = depth

α = energy correction factor

$V^2/2g$ = velocity head

Plate 6 shows a specific energy graph for a discharge q of 100 cubic feet per second (cfs) (two-dimensional flows). Each unit discharge has its own critical depth:

$$d_c = \left(\frac{q^2}{g}\right)^{1/3} \tag{2-9}$$

The development of this equation is given by pp 8-8 and 8-9 of Brater and King (1976). It may be noted that the critical depth occurs when the specific energy is at a minimum. Flow at a depth less than critical ($d < d_c$) will have velocities greater than critical ($V > V_c$), and the flow is described as rapid. Conversely, when $d > d_c$ and $V < V_c$, the flow is tranquil.

(b) It may be noted in Plate 6 that in the proximity of critical depth, a relatively large change of depth may occur with a very small variation of specific energy. Flow in this region is unstable and excessive wave action or undulations of the water surface may occur. Experiments by the US Army Engineer District (USAED), Los Angeles (1949), on a rectangular channel established criteria to avoid such instability, as follows:

Tranquil flow: $d > 1.1d_c$ or $F < 0.86$

Rapid flow: $d < 0.9d_c$ or $F > 1.13$

where F is the flow Froude number. The Los Angeles District model indicated prototype waves of appreciable height occur in the unstable range. However, there may be special cases where it would be more economical to provide sufficient wall height to confine the waves rather than modify the bottom slope.

(c) Flow conditions resulting with Froude numbers near 1.0 have been studied by Boussinesq and Fawer. The results of their studies pertaining to wave height with unstable flow have been summarized by Jaeger (1957, pp 127-131), including an expression for approximating the wave height. The subject is treated in more detail in paragraph 4-3d below. Determination of the critical depth instability region involves the proper selection of high and low resistance coefficients. This is demonstrated by the example shown in Plate 6 in which the depths are taken as normal depths and the hydraulic radii are equal to depths. Using the suggested equivalent roughness design values of $k = 0.007$ ft and $k = 0.002$ ft , bottom slope values of $S_o = 0.00179$ and $S_o = 0.00143$, respectively, are required at critical depth. For the criteria to avoid the region of instability ($0.9d_c < d < 1.1d_c$), use of the smaller k value for tranquil flow with the bottom slope adjusted so that $d \geq 1.1d_c$ will obviate increased wall heights for wave action. For rapid flow, use of the larger k value with the bottom slope adjusted so that $d \leq 0.9d_c$ will obviate increased wall heights should the actual surface be smoother. Thus, the importance of equivalent roughness and slope relative to stable flow is emphasized. These stability criteria should be observed in both uniform and nonuniform flow design.

(2) Pulsating rapid flow. Another type of flow instability occurs at Froude numbers substantially greater than 1. This type of flow is characterized by the formation of slugs particularly noticeable on steep slopes with shallow flow depth. A Manning's n for pulsating rapid flow can be computed from

$$\frac{0.0463R^{1/6}}{n} = 4.04 - \log_{10}\left(\frac{F}{F_s}\right)^{2/3} \tag{2-10}$$

The limiting Froude number F_s for use in this equation was derived by Escoffier and Boyd (1962) and is given by

$$F_s = \frac{\xi}{\sqrt{g}\ \zeta^{3/2}\ (1 + Z\zeta)} \tag{2-11}$$

where ξ , the flow function, is given by

$$\xi = \frac{Q}{b^{5/2}}$$

where Q is the total discharge and ζ , the depth-width ratio, is given by

$$\zeta = \frac{d}{b}$$

where b is the bottom width.

Plate 7 shows the curves for a rectangular channel and trapezoidal channels with side slopes Z of 1, 2, and 3.

(3) Varied flow profiles. The flow profiles discussed herein relate to prismatic channels or uniform cross section of boundary. A complete classification includes bottom slopes that are horizontal, less than critical, equal to critical, greater than critical, and adverse. However, the problems commonly encountered in design are mild slopes that are less than critical slope and steep slopes that are greater than critical slope. The three types of profiles in each of these two classes are illustrated in HDC 010-1. Chow (1959) gives a well-documented discussion of all classes of varied flow profiles. It should be noted that tranquil-flow profiles are computed proceeding upstream and rapid-flow profiles downstream. Flow profiles computed in the wrong direction result in divergences from the correct profile. Varied-flow computations used for general design should not pass through critical depth. Design procedures fall into two basic categories: uniform and nonuniform or varied flow. Many

graphs and tables have been published to facilitate computation of uniform flow. Brater and King (1976) have specially prepared tables for trapezoidal channels based on the Manning equation. HDC 610-1 through 610-4/1-1 give graphs that afford rapid solution for the normal depth in trapezoid channels. Nonuniform or varied flow in prismatic channels can be solved rapidly by use of the varied flow function. (It should be noted that different authors have used the terms "nonuniform" flow and "varied" flow to mean the same thing; "varied flow" is used in this manual.) Varied flow in nonprismatic channels, such as those with a gradually contracting or a gradually expanding cross section, is usually handled by "step methods." It should be noted that short, rapidly contracting or expanding cross sections are treated in this manual as transitions.

(a) Prismatic channels. A prismatic channel is characterized by unvarying cross section, constant bottom slope, and relatively straight alignment. There are three general methods of determining flow profiles in this type of channel: direct integration, direct step, and standard step, as discussed in Chow (1959, pp 252-268). The direct integration and direct step methods apply exclusively to prismatic channels, whereas the standard step method applies not only to prismatic channels but is the only method to be applied to nonprismatic channels. The direct integration method (with certain restrictions as to the constancy of hydraulic exponents) solves the varied flow equation to determine the length of reach between successive depths. Use is made of varied-flow-function tables to reduce the amount of computations. This method is not normally employed unless sufficient profiles and length of channel are involved to warrant the amount of precomputational preparation. The direct step method determines the length of reach between successive depths by solution of the energy and friction equations written for end sections of the reach. The standard step method is discussed in (b) below.

(b) Nonprismatic channels. When the cross section, alignment, and/or bottom slope changes along the channel, the standard step method (Chow 1959, p 265) is applied. This method determines the water-surface elevation (depth) at the reach extremity by successive approximations. Trial water-surface elevations are assumed until an elevation is found that satisfies the energy and friction equations written for the end sections of the reach. Cross sections for this method should, in general, be selected so that velocities are increasing or decreasing continuously throughout the reach. EM 1110-2-1409 contains further information on this method. Plate 8 shows a sample computation for a gradually contracting trapezoidal channel where both bottom width and side slope vary. Successive approximations of water-surface elevations are made until a balance of energy is obtained. Friction losses h_f are based on the Manning equation.

$$S_f = \frac{n^2V^2}{2.21R^{4/3}} = \frac{V^2}{C^2R} \qquad \text{(2-1 and 2-2 bis)}$$

For the sample computation a mild slope upstream and steep slope downstream of sta 682+40 have been assumed. Critical depth would occur in the vicinity of sta 682+40 and has been assumed as the starting condition. Initially, column 21 has the same value as column 10. The computations proceed downstream as the flow is rapid. The length of reach is chosen such that the change in velocity between the ends of the reach is less than 10 percent. The energy equation is balanced when column 21 checks column 10 for the trial water surface of column 5. Plate 9 repeats the computation, substituting k = 0.002 ft for n = 0.014 . For rough channel conditions

$$C = 32.6 \log_{10}\left(\frac{12.2R}{k}\right) \qquad \text{(2-6 bis)}$$

2-3. Flow Through Bridges

Bridge piers located in channels result in energy losses in the flow and create disturbances at the bridge section and in the channel sections immediately upstream and downstream. As bridge pier losses materially affect water-surface elevations in the vicinity of the bridge, their careful determination is important. Submergence of bridge members is not desirable.

a. Abutment losses. Bridge abutments should not extend into the flow area in rapid-flow channels. In tranquil-flow channels they should be so designed that the flow depth between abutments or between the abutment and an intermediate pier is greater than critical depth. The Bureau of Public Roads (BPR) (Bradley 1978) has published design charts for computing backwater for various abutment geometries and degrees of contraction. The design procedure and charts developed by BPR are recommended for use in channel designs involving bridge abutments. For preliminary designs, a step backwater computation using abrupt expansion and contraction head losses of 1.0 and 0.5, respectively, times the change in velocity head may be used. This method under the same circumstances may be applied to bridge openings containing piers.

b. Pier losses. Rapid, tranquil, or a combination of rapid- and tranquil-flow conditions may occur where only bridge piers are located in the flow area. Flow through bridge piers for this condition is classified as class A, B, or C, according to the depth of flow in relation to critical depth occurring upstream, between piers, and downstream. Plate 10 is a graphic description of these classes, which are discussed below. Plate 11 is useful in determining the class of flow in rectangular channels.

(1) Class A flow (energy method). Chow (1959, paragraph 17-10) presents a discussion and several energy loss formulae with appropriate coefficients that may be used for computing bridge pier losses for tranquil flow (class A). While the momentum method presented below may also be used for class A flow, the energy method usually gives better results.

(2) Classes B and C flows (momentum method).

(a) A graph (example shown in Plate 12) constructed from the equation proposed by Koch and Carstanjen (Chow 1959) and based on the momentum relation can be used for determining graphically the flow classification at constrictions due to bridge piers. In addition, the graph can be used for estimating unknown flow depths. A summary of the equation derivation follows.

(b) In a given channel section the momentum per unit time of the flow can be expressed by

$$M = \beta \left(\frac{\gamma Q V}{g} \right) \tag{2-12}$$

where

M = momentum per unit time, pounds (lb) (from pounds-second per second (lb-sec/sec))

β = momentum correction coefficient

γ = specific weight of water, pounds per cubic foot (pcf)

Q = total discharge, cfs

V = average channel velocity, feet per second (fps)

g = acceleration of gravity, ft/sec^2

In Equation 2-12 β is generally assumed to be equal to 1.0. Since

$$Q = AV \tag{2-13}$$

Equation 12 can be written

$$M = \frac{\gamma Q^2}{gA} \tag{2-14}$$

(c) The total hydrostatic force m (in pounds) in the channel section can be expressed as

$$m = \gamma \bar{y} A \tag{2-15}$$

where $\bar{y}$ is the distance from the water surface to the center of gravity (centroid) of the flow section.

(d) Combining Equations 14 and 15 results in

$$m + M = \gamma \bar{y} A + \frac{\gamma Q^2}{gA} \tag{2-16}$$

By the momentum principle in an unconstricted channel

$$m_a + \frac{\gamma Q^2}{gA_a} = m_b + \frac{\gamma Q^2}{gA_b} \tag{2-17}$$

where m_a and m_b are the total hydrostatic forces of water in the upstream and downstream sections, respectively, lb.

(e) Based on experiments under all conditions of open-channel flow where the channel was constricted by short, flat surfaces perpendicular to the flow, such as with bridge piers, Koch and Carstanjen (Koch 1926) found that the upstream momentum force had to be reduced by $(A_p/A_1)(\gamma Q^2/gA_1)$ to balance the total force in the constriction.

(f) Equating the summation of the external forces above and below the structures with those within the contracted section yields

$$m_1 + \frac{\gamma Q^2}{gA_1} - \left[\left(\frac{A_p}{A_1}\right)\left(\frac{\gamma Q^2}{gA_1}\right)\right] = m_2 + m_p + \frac{\gamma Q^2}{gA_2} \qquad (2\text{-}18)$$

and

$$m_2 + m_p + \frac{\gamma Q^2}{gA_2} = m_3 + \frac{\gamma Q^2}{gA_3} \qquad (2\text{-}19)$$

Combining these equations results in

$$m_1 + \frac{\gamma Q^2}{gA_1} - \left[\left(\frac{A_p}{A_1}\right)\left(\frac{\gamma Q^2}{gA_1}\right)\right] = m_2 + m_p + \frac{\gamma Q^2}{gA_2} = m_3 + \frac{\gamma Q^2}{gA_3} \qquad (2\text{-}20)$$

This reduces to the Koch-Carstanjen equation

$$m_1 - m_p + \frac{\gamma Q^2}{gA_1^2}(A_1 - A_p) = m_2 + \frac{\gamma Q^2}{gA_2} = m_3 - m_p + \frac{\gamma Q^2}{gA_3} \qquad (2\text{-}21)$$

where

γ = specific weight of water, pounds per cubic foot (pcf)

Q = total discharge, cfs

m_1 = total hydrostatic force of water in upstream section, lb

m_2 = total hydrostatic force of water in pier section, lb

m_3 = total hydrostatic force of water in downstream section, lb

m_p = total hydrostatic force of water on pier ends, lb

A_1 = cross-sectional area of upstream channel, square feet, ft^2

A_2 = cross-sectional area of channel within pier section, ft^2

A_3 = cross-sectional area of downstream channel, ft^2

A_p = cross-sectional area of pier obstruction, ft^2

(g) Curves based on the Koch-Carstanjen equation (Equation 2-21) are illustrated in Plate 12a. The resulting flow profiles are shown in Plate 12b. The necessary computations for developing the curves are shown in Plate 13. The downstream depth is usually known for tranquil-flow channels and is greater than critical depth. It therefore plots on the upper branch of curve III in Plate 12a. If this depth *A* is to the right of (greater force than) the minimum force value *B* of curve II, the flow is class A and the upstream design depth *C* is read on curve I immediately above point *A*. In this case, the upstream depth is controlled by the downstream depth *A* plus the pier contraction and expansion losses. However, if the downstream depth *D* plots on the upper branch of curve III to the left of (less force than) point *B*, the upstream design depth *E* is that of curve I immediately above point *B*, and critical depth within the pier section *B* is the control. The downstream design depth *F* now is that given by curve III immediately below point *E*. A varied flow computation in a downstream direction is required to determine the location where downstream channel conditions effect the depth *D*.

(h) In rapid-flow channels, the flow depth upstream of any pier effect is usually known. This depth is less than critical depth and therefore plots on the lower branch of curve I. If this depth *G* is located on curve I to the right of point *B*, the flow is class C. The downstream design depth *H* and the design flow depth within the pier section *I* are read on curves III and II, respectively, immediately above depth *G*. A varied flow computation is required to determine the location where downstream channel conditions again control the depth. However, if the unaffected upstream rapid-flow depth *J* plots on the lower branch of curve I to the left of point *B*, the design upstream depth *K* is read on curve I immediately above point *B*. The design downstream depth *L* is read on curve II immediately below point *B*. In this case, class B flow results with a hydraulic jump between depths *J* and *K* (Plate 12b) upstream of the pier as controlled by critical depth within the pier section *B*. A varied flow computation is again required to determine the location where downstream channel conditions control the flow depth.

(3) Design charts, rectangular sections. A graphic solution for class A flow in rectangular channels, developed by USAED, Los Angeles (1939), and published as HDC 010-6/2, is reproduced in Plate 14. The drop in water surface H_3 in terms of critical depth is presented as a function of the downstream depth d_3 and critical depth in the unobstructed channel. Separate curves are given for channel contraction ratios of between 0.02 and 0.30. In rectangular channels, α is the horizontal contraction ratio. The basic graph is for round nose piers. The insert graph provides correction factors (γ) for other pier shapes. Use of the chart is illustrated in Plate 15. Plate 16 (HDC 010- 6/3) presents the USAED, Los Angeles, (1939), solution for class B flow using the momentum method. Plate 17 (HDC 010-6/4) presents the USAED, Chicago, solution for class B flow by the energy method. The use of these charts for rectangular channel sections is shown in Plate 15.

c. Bridge pier extension. Upstream pier extensions are frequently used to reduce flow disturbance caused by bridge piers and to minimize collection of debris on pier noses. In addition, it is often necessary and economical to make use of existing bridge structures in designing flood channels. In some instances there is insufficient vertical clearance under these structures to accommodate the design flow. With class B flow, the maximum flow depth occurs at the upstream end of the pier and the critical depth occurs within the constriction. Field observations and model studies by USAED, Los Angeles (1939), indicate that the minimum depth within the constricted area usually occurs 15 to 25 ft downstream from the upstream end of the pier. Pier extensions are used to effect an upstream movement of the control section, which results in a depth reduction in the flow as it enters the constricted pier section. The use of bridge pier extensions to accomplish this is illustrated in USAED, Los Angeles (1943), and USAEWES (1957). The general statements relative to bridge pier extensions for class B flow also apply to class C flow. However, in the latter case, the degree and extent of the disturbances are much more severe than with class B flow. Excellent illustrations of the use of bridge pier extensions in high-velocity channels are given in USAED, Los Angeles (1943), and USAED, Walla Walla (1960). The bridge pier extension geometry shown in Plate 18 was developed by USAED, Los Angeles, and pier extensions of this design have been found to perform satisfactorily.

d. Model studies. Where flow conditions at bridge piers are affected by severe changes in channel geometry and alignment, bridge abutments, or multiple bridge crossings, consideration should be given to obtaining the design flow profile from a hydraulic model study.

2-4. Transitions

a. General. Transitions should be designed to accomplish the necessary change in cross section with as little flow disturbance as is consistent with economy. In tranquil flow, the head loss produced by the transition is most important as it is reflected as increased upstream stages. In rapid flow, standing waves produced by changes of direction are of great concern in and downstream from the transition. Streamlined transitions reduce head losses and standing waves. As transition construction costs exceed those of uniform channel cross section and tend to increase with the degree of streamlining, alternative transition designs, their costs, and the incremental channel costs due to head losses and/or standing waves should be assessed.

b. Types. The three most common types of transitions connecting trapezoidal and rectangular channels are cylindrical quadrant, warped, and wedge, as shown in Plate 19. For comparable design, the wedge-type transition, although easier to construct, should be longer than the warped because of the miter bends between channel and transition faces. Warped and wedge types can be used generally for expansions or contractions.

(1) Tranquil flow. Each of these three transition types may be used for tranquil flow in either direction. The cylindrical quadrant is used for expansions from rectangular to trapezoidal section and for contractions from trapezoidal to rectangular section. An abrupt or straight-line transition as well as the quadrant transition can be used in rectangular channels.

(2) Rapid flow. The cylindrical quadrant is used for transitions from tranquil flow in a trapezoidal section to rapid flow in a rectangular section. The straight-line transition is used for rectangular sections with rapid flow. Specially designed curved expansions (c(2)(b) below) are required for rapid flow in rectangular channels.

c. Design.

(1) Tranquil flow. Plate 20 gives dimensions of plane surface (wedge type) transitions from rectangular to trapezoidal cross section having side slopes of 1 on 2; 1 on 2.5, and 1 on 3. In accordance with the recommendations of Winkel (1951) the maximum change in flow line has been limited to 6.0 degrees (deg). Water-surface profiles should be determined by step computations with less than 20 percent (less than 10 percent in important instances) change in velocity between steps. Adjustments in the transition should be made, if necessary, to obtain a water-surface profile that is as nearly straight as practicable.

(2) Rapid flow. In rapid flow, stationary waves result with changes in channel alignment. These disturbances may necessitate increased wall height, thereby appreciably increasing construction costs. USAED, Los Angeles, uses the criterion in Table 2-2 for the design of straight-line or wedge-type transitions to confine flow disturbances within the normal channel freeboard allowance:

Table 2-2
Recommended Convergence and Divergence Transition Rates

Mean channel velocity, fps	Wall flare for each wall (horizontal to longitudinal)
10-15	1:10
15-30	1:15
30-40	1:20

(a) Rectangular contractions. Ippen (1950), Ippen and Dawson (1951), and Ippen and Harleman (1956) applied the wave theory to the design of rectangular channel transitions for rapid flow and developed the following equations for computing flow depths in and downstream from the convergence:

$$\tan\theta = \frac{\tan\beta_1\left(\sqrt{1 + 8F_1^2\sin^2\beta_1} - 3\right)}{2\tan^2\beta_1 + \sqrt{1 + 8F_1^2\sin^2\beta_1} - 1} \qquad (2\text{-}22)$$

$$\frac{y_2}{y_1} = \frac{1}{2}\left(\sqrt{1 + 8F_1^2\sin^2\beta_1} - 1\right) \qquad (2\text{-}23)$$

and

$$F_2^2 = \frac{y_1}{y_2}\left[F_1^2 - \frac{1}{2}\frac{y_1}{y_2}\left(\frac{y_2}{y_1} - 1\right)\left(\frac{y_2}{y_1} + 1\right)^2\right] \qquad (2\text{-}24)$$

where

θ = wall deflection angle

F = Froude number

β = wave front angle

y = flow depth

The subscripts 1, 2, and 3 refer to the flow areas indicated on the sketches in Plate 21. For straight-line convergence (Plate 21b), the maximum flow disturbance results when the initial wave front intersection, point *B*, occurs at the downstream transition *CC*'. When the reflected waves *BD* and *BD*' intersect the channel walls below or above section *CC*', diamond-shaped cross waves develop in the channel. However, the change in wall alignment at section *CC*' results in negative wave disturbances that should tend to decrease the downstream effects of positive wave fronts. This should result in somewhat lower depths where the waves meet the downstream walls. The minimum disturbance occurs when the reflected waves *BD* and *BD*' meet the channel walls at section *CC*'. This, theoretically, results in the flow filaments again becoming parallel to the channel center line. If the reflected waves meet the walls upstream from section *CC*', the waves would be deflected again with a resulting increase in depth. Graphic plots of Equations 2-22 through 2-24 have been published (Ippen 1950, Ippen and Dawson 1951, and

Ippen and Harleman 1956). Plate 22 presents design curves based on these equations. The extent of the curves has been limited to flow conditions normally occurring in rapid-flow flood control channels. The required length of the transition is a function of the wall deflection angle θ and the channel contraction $b_1 - b_3$, or

$$L = \frac{b_1 - b_3}{2 \tan \theta} \tag{2-25}$$

where

b_1 = upstream channel width, ft

b_3 = downstream channel width, ft

The theory indicates that the surface disturbances are minimized when $L = L_1 + L_2$ (Plate 21). The equations for L_1 and L_2 are

$$L_1 = \frac{b_1}{2 \tan \beta_1} \tag{2-26}$$

and

$$L_2 = \frac{b_3}{2 \tan (\beta_2 - \theta)} \tag{2-27}$$

The correct transition design for a given change in channel width and Froude number involves selection of a value of θ so that $L = L_1 + L_2$. A computation illustrating the design procedure is given in Plate 23.

(b) Rectangular expansions. In channel expansions the changes in flow direction take place gradually in contrast to the steep wave front associated with contractions. In 1951, Rouse, Bhoota, and Hsu (1951) published the results of a study of expanding jets on a horizontal floor. A graphical method of characteristics, described in Ippen (1951), was used for the theoretical development of flow depth contours. These results were verified experimentally. The following equation based on theoretical and experimental studies was found to give the most satisfactory boundary shapes for the expansion of a high-velocity jet on a horizontal floor.

$$\frac{Z}{b_1} = \frac{1}{2} \left(\frac{X}{b_1 F_1} \right)^{3/2} + \frac{1}{2} \tag{2-28}$$

where

Z = transverse distance from channel center line

b_1 = approach channel width

X = longitudinal distance from beginning of expansion

F_1 = approach flow Froude number

Equation 2-28 is for an infinitely wide expansion. Optimum design of expansions for rapid flow necessitates control of wall curvature so that the negative waves generated by the upstream convex wall are compensated for by positive waves formed by the downstream concave wall. In this manner, the flow is restored to uniformity where it enters the downstream channel. A typical design of a channel expansion is shown in Plate 24b. Plate 24a reproduces generalized design curves presented in Rouse, Bhoota, and Hsu (1951). It is to be noted that the convex wall curve equation is appreciably less severe than that indicated by Equation 2-28. Equations for laying out the transition and a definition sketch are given in Plate 24b. The data given in Plate 24 should be adequate for preliminary design. In cases where the wave effects are critical, the design should be model tested. Laboratory experiments based on the generalized curves have indicated that the downstream channel depths may be appreciably in excess of those indicated by the simple wave theory. The simple wave theory can be applied to the design of straight-line transitions. An illustration of the computation procedure is given on pages 9-10 through 9-12 of Brater and King (1976). It is to be noted that this computation does not include any wave effects reflected from one sidewall to the other. Also, an abrupt positive wave exists where the expanding wall intersects the downstream channel wall. Application of this method of characteristics is illustrated on pages 9-12 through 9-16 of Brater and King (1976).

(c) Nonrectangular transitions. The necessary techniques for applying the wave theory to channel transitions involving both rectangular and trapezoidal sections have not been developed, and generalized design curves are not available. Limited tests on straight-line and warped-wall channel transitions for trapezoidal to rectangular sections and for rectangular to trapezoidal sections have been made at Pennsylvania State University (Blue and Shulits 1964). Tests were limited to three different transition shapes for Froude numbers of 1.2 to 3.2. Each shape was tested for five different transition lengths. The trapezoidal channel invert was 0.75 ft wide. The rectangular channel was 1.071 ft wide. Generalized design curves were not developed. However, the study results should be useful as design guides.

(3) Rapid to tranquil flow.

(a) The design of rapid-flow channels may require the use of transitions effecting flow transformation from rapid to tranquil flow. Such transitions normally involve channel expansions in which the channel shape changes from rectangular to trapezoidal.

(b) Channel expansions in which the flow changes from rapid to tranquil are normally of the wedge type. The flow transformation can be accomplished by means of the abrupt hydraulic jump or by a gradual flow change involving an undular-type jump. In either case, it is necessary that the flow transformation be contained in the transition section. The use of a stilling-basin type of transition to stabilize the hydraulic jump is illustrated in USAED, Los Angeles (1961) and USAEWES (1962). A typical example of this type of transition is given in Plate 25.

(c) USAED, Los Angeles (1958, 1961, 1962) has designed and model tested a number of transitions transforming rapid flow in rectangular channels to tranquil flow in trapezoidal channels without the occurrence of an abrupt hydraulic jump. The high-velocity jet from the rectangular channel is expanded in the transition by means of lateral and boundary roughness control in such a manner that an undular-type jump occurs in the downstream reach of the transition. Plate 26 illustrates a typical design developed through model tests.

d. Transition losses.

(1) Tranquil flow. Transitions for tranquil flow are designed to effect minimum energy losses consistent with economy of construction. Transition losses are normally computed using the energy equation and are expressed in terms of the change in velocity head Δh_v from upstream to downstream of the transition. The head loss h_1 between cross sections in the step computation may be expressed as

$$h_1 = C_c \Delta h_v \tag{2-29}$$

for contractions and as

$$h_1 = C_e \Delta h_v \tag{2-30}$$

where

C_c = contraction coefficient

C_e = expansion coefficient

for expansions. Equations 2-29 and 2-30 have been obtained and published (Chow 1959, Brater and King 1976, US Bureau of Reclamation (USBR) 1967). The values in Table 2-3 are generally accepted for design purposes.

Table 2-3
Transition Loss Coefficients

Transition Type	C_c	C_e	Source
Warped	0.10	0.20	Chow 1959, Brater and King 1976
Cylindrical Quadrant	0.15	0.20	Chow 1959
Wedge	0.30	0.50	USBR 1967
Straight Line	0.30	0.50	Chow 1959
Square End	0.30	0.75	Chow 1959

(2) Rapid flow. Transition losses may be estimated for rapid-flow conditions from the information supplied in (1) above. However, the effects of standing waves and other factors discussed in c(2) above make exact

determinations of losses difficult. Model tests should be considered for important rapid-flow transitions.

2-5. Flow in Curved Channels

a. General.

(1) The so-called centrifugal force caused by flow around a curve results in a rise in the water surface on the outside wall and a depression of the surface along the inside wall. This phenomenon is called superelevation. In addition, curved channels tend to create secondary flows (helicoidal motion) that may persist for many channel widths downstream. The shifting of the maximum velocity from the channel center line may cause a disturbing influence downstream. The latter two phenomena could lead to serious local scour and deposition or poor performance of a downstream structure. There may also be a tendency toward separation near the inner wall, especially for very sharp bends. Because of the complicated nature of curvilinear flow, the amount of channel alignment curvature should be kept to a minimum consistent with other design requirements.

(2) The required amount of superelevation is usually small for the channel size and curvature commonly used in the design of tranquil-flow channels. The main problem in channels designed for rapid flow is standing waves generated in simple curves. These waves not only affect the curved flow region but exist over long distances downstream. The total rise in water surface for rapid flow has been found experimentally to be about twice that for tranquil flow.

(3) Generally, the most economical design for rapid flow in a curved channel results when wave effects are reduced as much as practical and wall heights are kept to a minimum. Channel design for rapid flow usually involves low rates of channel curvature, the use of spiral transitions with circular curves, and consideration of invert banking.

b. Superelevation. The equation for the transverse water-surface slope around a curve can be obtained by balancing outward centrifugal and gravitational forces (Woodward and Posey 1941). If concentric flow is assumed where the mean velocity occurs around the curve, the following equation is obtained

$$\Delta y = C \frac{V^2 W}{gr} \tag{2-31}$$

where

Δy = rise in water surface between a theoretical level water surface at the center line and outside water-surface elevation (superelevation)

C = coefficient (see Table 2-4)

V = mean channel velocity

W = channel width at elevation of center-line water surface

g = acceleration of gravity

r = radius of channel center-line curvature

Use of the coefficient C in Equation 2-31 allows computation of the total rise in water surface due to superelevation and standing waves for the conditions listed in Table 2-4. If the total rise in water surface (superelevation plus surface disturbances) is less than 0.5 ft, the normally determined channel freeboard (paragraph 2-6 below) should be adequate. No special treatment such as increased wall heights or invert banking and spiral transitions is required.

Table 2-4
Superelevation Formula Coefficients

Flow Type	Channel Cross Section	Type of Curve	Value of C
Tranquil	Rectangular	Simple Circular	0.5
Tranquil	Trapezoidal	Simple Circular	0.5
Rapid	Rectangular	Simple Circular	1.0
Rapid	Trapezoidal	Simple Circular	1.0
Rapid	Rectangular	Spiral Transitions	0.5
Rapid	Tapezoidal	Spiral Transitions	1.0
Rapid	Rectangular	Spiral Banked	0.5

(1) Tranquil flow. The amount of superelevation in tranquil flow around curves is small for the normal channel size and curvature used in design. No special treatment of curves such as spirals or banking is usually necessary. Increasing the wall height on the outside of the curve to contain the superelevation is usually the most economical remedial measure. Wall heights should be increased by Δy over the full length of curvature. Wall heights on the inside of the channel curve should be held

to the straight channel height because of wave action on the inside of curves.

(2) Rapid flow. The disturbances caused by rapid flow in simple curves not only affect the flow in the curve, but persist for many channel widths downstream. The cross waves generated at the beginning of a simple curve may be reinforced by other cross waves generated farther downstream. This could happen at the end of the curve or within another curve, provided the upstream and downstream waves are in phase. Wall heights should be increased by the amount of superelevation, not only in the simple curve, but for a considerable distance downstream. A detailed analysis of standing waves in simple curves is given in Ippen (1950). Rapid-flow conditions are improved in curves by the provision of spiral transition curves with or without a banked invert, by dividing walls to reduce the channel width, or by invert sills located in the curve. Both the dividing wall and sill treatments require structures in the flow; these structures create debris problems and, therefore, are not generally used.

(a) Spiral transition curves. For channels in which surface disturbances need to be minimized, spiral transition curves should be used. The gradual increase in wall deflection angles of these curves results in minimum wave heights. Two spiral curves are provided, one upstream and one downstream of the central circular curve. The minimum length of spirals for unbanked curves should be determined by (see Douma, p 392, in Ippen and Dawson 1951)

$$L_s = 1.82 \frac{VW}{\sqrt{gy}} \qquad (2\text{-}32)$$

where y is the straight channel flow depth.

(b) Spiral-banked curves. For rectangular channels, the invert should be banked by rotating the bottom in transverse sections about the channel center line. Spirals are used upstream and downstream of the central curve with the banking being accomplished gradually over the length of the spiral. The maximum amount of banking or difference between inside and outside invert elevations in the circular curve is equal to twice the superelevation given by Equation 2-31. The invert along the inside wall is depressed by Δy below the center-line elevation and the invert along the outside wall is raised by a like amount. Wall heights are usually designed to be equal on both sides of the banked curves and no allowance needs to be made for superelevation around the curve. The minimum length of spiral should be 30 times the amount of superelevation (Δy) (USAED, Los Angeles, 1950).

$$L_s = 30\Delta y \qquad (2\text{-}33)$$

The detailed design of spiral curves is given in Appendix D. A computer program for superelevation and curve layout is included. Banked inverts are not used in trapezoidal channels because of design complexities and because it is more economical to provide additional freeboard for the moderate amount of superelevation that usually occurs in this type of channel.

c. Limiting curvature. Laboratory experiments and field experience have demonstrated that the helicoidal flow, velocity distribution distortion, and separation around curves can be minimized by properly proportioning channel curvature. Woodward (1920) recommends that the curve radius be greater than 2.5 times the channel width. From experiments by Shukry (1950) the radius of curvature should be equal to or greater than 3.0 times the channel width to minimize helicoidal flow.

(1) Tranquil flow. For design purposes a ratio of radius to width of 3 or greater is suggested for tranquil flow.

(2) Rapid flow. Large waves are generated by rapid flow in simple curves. Therefore a much smaller rate of change of curvature is required than for tranquil flow. A 1969 study by USAED, Los Angeles (1972), of as-built structures shows that curves with spiral transitions, with or without banked inverts, have been constructed with radii not less than

$$r_{\min} = \frac{4V^2W}{gy} \qquad (2\text{-}34)$$

where

r_{min} = minimum radius of channel curve center line

V = average channel velocity

W = channel width at water surface

y = flow depth

The amount of superelevation required for spiral-banked curves (b above) is given by

$$\Delta y = C \frac{V^2 W}{gr} \qquad (2\text{-}35)$$

However, this study indicates that the maximum allowable superelevation compatible with Equation 2-34 is

$$2\Delta y = W \tan 10 = 0.18W \qquad (2\text{-}36)$$

or

$$\Delta y = 0.09W$$

d. Bend loss. There has been no complete, systematic study of head losses in channel bends. Data by Shukry (1950), Raju (1937), and Bagnold (1960) suggest that the increased resistance loss over and above that attributable to an equivalent straight channel is very small for values of $r/W > 3.0$. For very sinuous channels, it may be necessary to increase friction losses used in design. Based on tests in the Tiger Creek Flume, Scobey (1933) recommended that Manning's n be increased by 0.001 for each 20 deg of curvature per 100 ft of channel, up to a maximum increase of about 0.003. The small increase in resistance due to curvature found by Scobey was substantiated by the USBR field tests (Tilp and Scrivner 1964) for $r/W > 4$. Recent experiments have indicated that the channel bend loss is also a function of Froude number (Rouse 1965). According to experiments by Hayat (Rouse 1965), the free surface waves produced by flow in a bend can cause an increase in resistance.

2-6. Special Considerations

a. Freeboard.

(1) The freeboard of a channel is the vertical distance measured from the design water surface to the top of the channel wall or levee. Freeboard is provided to ensure that the desired degree of protection will not be reduced by unaccounted factors. These might include erratic hydrologic phenomena; future development of urban areas; unforeseen embankment settlement; the accumulation of silt, trash, and debris; aquatic or other growth in the channels; and variation of resistance or other coefficients from those assumed in design.

(2) Local regions where water- surface elevations are difficult to determine may require special consideration. Some examples are locations in or near channel curves, hydraulic jumps, bridge piers, transitions and drop structures, major junctions, and local storm inflow structures. As these regions are subject to wave-action uncertainties in water-surface computations and possible overtopping of walls, especially for rapid flow, conservative freeboard allowances should be used. The backwater effect at bridge piers may be especially critical if debris accumulation is a problem.

(3) The amount of freeboard cannot be fixed by a single, widely applicable formula. It depends in large part on the size and shape of channel, type of channel lining, consequences of damage resulting from overtopping, and velocity and depth of flow. The following approximate freeboard allowances are generally considered to be satisfactory: 2 ft in rectangular cross sections and 2.5 ft in trapezoidal sections for concrete-lined channels; 2.5 ft for riprap channels; and 3 ft for earth levees. The freeboard for riprap and earth channels may be reduced somewhat because of the reduced hazard when the top of the riprap or earth channels is below natural ground levels. It is usually economical to vary concrete wall heights by 0.5-ft increments to facilitate reuse of forms on rectangular channels and trapezoidal sections constructed by channel pavers.

(4) Freeboard allowances should be checked by computations or model tests to determine the additional discharge that could be confined within the freeboard allowance. If necessary, adjustments in freeboard should be made along either or both banks to ensure that the freeboard allowance provides the same degree of protection against overtopping along the channel.

b. Sediment transport. Flood control channels with tranquil flow usually have protected banks but unprotected inverts. In addition to reasons of economy, it is sometimes desirable to use the channel streambed to percolate water into underground aquifers (USAED, Los Angeles, 1963). The design of a channel with unprotected inverts and protected banks requires the determination of the depth of the bank protection below the invert in regions where bed scour may occur. Levee heights may depend on the amount of sediment that may deposit in the channel. The design of such channels requires estimates of sediment transport to predict channel conditions under given flow and sediment characteristics. The subject of sediment transport in alluvial channels and design of canals has been ably presented by Leliavsky (1955). Fundamental information on bed-load equations and their background with examples of use in channel design is given in Rouse (1950) (see pp 769-857). An excellent review with an extensive bibliography is available (Chien 1956). This review includes the generally accepted Einstein approach to sediment transport. A comparative treatment of the many bed-load equations (Vanoni, Brooks, and Kennedy 1961) with field data indicates that no one formula is conclusively better than any other and that the accuracy of prediction is about ±100 percent. A recent paper by Colby (1964b) proposes a simple, direct method of empirically correlating bed- load discharge with mean channel velocity at various flow depths and median grain size diameters. This procedure is adopted herein for rough estimates of bed-load movement in flood control channels.

c. Design curves. Plate 27 gives curves of bed-load discharge versus channel velocity for three depths of flow and four sediment sizes. The basic ranges of depths and velocities have been extrapolated and interpolated from the curves presented in Colby (1964a) for use in flood control channel design. Corrections for water temperature and concentration of fine sediment (Colby 1964a) are not included because of their small influence. The curves in Plate 27 should be applicable for estimating bed-load discharge in channels having geologic and hydraulic characteristics similar to those in the channels from which the basic data were obtained. The curves in this plate can also be used to estimate the relative effects of a change in channel characteristics on bed-load movement. For example, the effect of a series of check dams or drop structures that are provided to decrease channel slope would be reflected in the hydraulic characteristics by decreasing the channel velocity. The curves could then be used to estimate the decrease in sediment load. The curves can also be used to approximate the equilibrium sediment discharge. If the supply of sediment from upstream sources is less than the sediment discharge computed by the rating curves, the approximate amount of streambed scour can be estimated from the curves. Similarly, deposition will occur if the sediment supply is greater than the sediment discharge indicated by the rating curves. An example of this is a large sediment load from a small side channel that causes deposition in a major flood channel. If the location of sediment deposition is to be controlled, the estimated size of a sediment detention facility can be approximated using the curves. An example of the use of a sediment discharge equation in channel design is given in USAED, Los Angeles (1963).

2-7. Stable Channels

a. General.

(1) The design of stable channels requires that the channel be in material or lined with material capable of resisting the scouring forces of the flow. Channel armoring is required if these forces are greater than those that the bed and bank material can resist. The basic principles of stable channel design have been presented by Lane (1955) and expanded and modified by Terrell and Borland (1958) and Carlson and Miller (1956). An outline of the method of channel design to resist scouring forces has been given in Simons (1957). The most common type of channel instability encountered in flood control design is scouring of the bed and banks. This results from relatively large discharges, steep channel slopes, and normally limited channel right-of-way widths. These factors frequently require the use of protective revetment to prevent scouring.

(2) While clay and silt are fairly resistant to scour, especially if covered with vegetation, it is necessary to provide channel revetment when tractive forces are sufficiently high to cause erosion of channels in fine material. Little is known about the resistance of clay and silt to erosion as particles in this size range are influenced to a large extent by cohesive forces. A summary of some of the effects is given by the Task Committee on Preparation of Sedimentation Manual (1966). Suggested maximum limiting average channel velocities for noncohesive materials are listed in c below and plotted in Plate 28.

b. Prevention of scour. Scour and deposition occur most commonly when particle sizes range from fine sand to gravel, i.e., from about 0.1 mm through 50 mm (Plate 28). Erosion of sands in the lower range of sizes is especially critical as the sand particle weight is small, there is no cohesion between grains, and there is usually little vegetation along the channel. This particle size range comprises the majority of the bed and suspended load in many streams. Paragraph 2-6 above discusses sediment movement and presents a sediment rating curve as a guide to predicting channel stability.

c. Permissible velocity and shear. The permissible velocity and shear for a nonerodible channel should be somewhat less than the critical velocity or shear that will erode the channel. The adoption of maximum permissible velocities that are used in the design of channels has been widely accepted since publication of a table of values by Fortier and Scobey (1926). The latest information on critical scour velocities is given by the Task Committee on Preparation of Sedimentation Manual (1966). Table 2-5 gives a set of permissible velocities that can be used as a guide to design nonscouring flood control channels. Lane (1955) presents curves showing permissible channel shear stress to be used for design, and the Soil Conservation Service (1954) presents information on grass-lined channels. Departures from suggested permissible velocity or shear values should be based on reliable field experience or laboratory tests. Channels whose velocities and/or shear exceed permissible values will require paving or bank revetment. The permissible values of velocity and/or shear should be determined so that damage exceeding normal maintenance will not result from any flood that could be reasonably expected to occur during the service life of the channel.

Table 2-5
Suggested Maximum Permissible Mean Channel Velocities

Channel Material	Mean Channel Velocity, fps
Fine Sand	2.0
Coarse Sand	4.0
Fine Gravel[1]	6.0
Earth	
Sandy Silt	2.0
Silt Clay	3.5
Clay	6.0
Grass-lined Earth (slopes less than 5%)[2]	
Bermuda Grass	
Sandy Silt	6.0
Silt Clay	8.0
Kentucky Blue Grass	
Sandy Silt	5.0
Silt Clay	7.0
Poor Rock (usually sedimentary)	10.0
Soft Sandstone	8.0
Soft Shale	3.5
Good Rock (usually igneous or hard metamorphic)	20.0

Notes:
1. For particles larger than fine gravel (about 20 millimetres (mm) = 3/4 in.), see Plates 29 and 30.
2. Keep velocities less than 5.0 fps unless good cover and proper maintenance can be obtained.